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Design of Safe Slopes After Failure During an Earthquake

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SYNOPSIS : Four slope slides took place, during a major earthquake, on the slopes of hill that has a paper mill complex on its top. The subsoil condition and engineering parameters for the site were evaluated at the time of construction of this complex and again after the earthquake for the purpose of designing safe slopes. However, both times the variation in the numerical values of shear parameters obtained by different tests was very wide and it was difficult to arrive at some conclusion. Therefore, on the basis of failure surface geometries, these was assessed by back analysis and design of safe slope carried out.

INTRODUCTION

A major earthquake measuring 7.0 on Richter Scale occurred in North-East region of India on August 6, 1988 causing loss to life and property. In this earthquake four major land slides took place on the slopes of hill that has Nagaland Pulp and Paper Mill on its top.This site is located in a highly seismic region and in a heavy precipitation area in Nagaland having yearly rainfall of the order of 250 cm. This paper presents the analysis of these slides and and design of the safe slopes for the site.

The problem of stability of slopes around the Pulp and paper mill complex is at four sections as shown in Fig. 1. The slopes at sections 1-1, 2-2, 3-3, 4-4 were 26.5°, 25°, 28.6°, and 33° respectively before earthquake whereas these slopes changed to 20°, 23°, 24°, and 32° due to the earthquake.

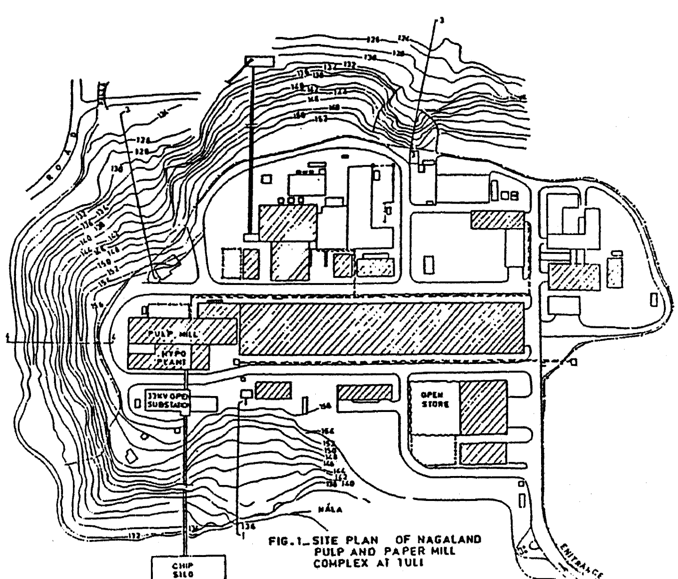

FIG.1_SITE PLAN OF NAGALAND PULP AND PAPER MILL COMPLEX AT TULI

SITE SUB-SOIL CONDITION

Initially, soil investigations at the site were carried out at the time of construction of the complex in 1973-74. The aim was to determine the soil profil at the proposed site as well as its variation and to evaluate the variation of insitu strength with depth over the area. It is reported that dense layer of silty sand underlain by about 10 ft. thick very dense and hard glacial conglomerate consisting of 8 to 12 inches size red sandstone boulders in a matrix of red to white silty clay sand exists. This layer is in turn underlain by very dense golden brownish yellow sand and very hard bluish grey sandy silt. Below the compacted layer is again reddish brown silty sand with presence of sandstone boulders.

In 1989 bore holes up to depth of 30 metre were made at the site. It is seen from the borelog data that the soil consists of mainly sandy silt, silty sand, clayey silt and silty clay with occasionally encountered layers of pebbles and gravels. The standard penetration test counts vary from 2 to more than 100. From these results it is inferred that the soil is mostly silt with sand.

The strength properties of the soil at site were evaluated earlier by conducting direct shear test, triaxial test for UU, CU, CD conditions. These results are given in Table - 1.A & 1.B.

The results show that for direct shear test, friction angle of the soil varies from 19° to 35° as per earlier test and 23° to 29° as per test done recently. The cohesion parameter vary from 0.0 to 0.15 kg/cm^2 & 0.05 to 0.12 kg/cm^2 respectively. As per earlier report the UU triaxial tests indicate value of angle of friction from 0° to 3° and cohesion 0.10 to 0.75 kg/cm^2. The CU test gives these as 5° to 10° and 0.10 to 0.15 kg/cm. The CD test gives 6° to 13° and 0.05 to 0.15 kg/cm^2. The triaxial test done in 1989 indicate the variation of ϕ from 10° to 17° and of C vary from 0.018 to 0.11 kg/cm^2.

TABLE - 1.A

RESULTS OF SHEAR PARAMETERS C & ϕ

SAMPLE LOCATION	DRY DENSITY (gm/cc)	MOISTURE CONTENT %	DIRECT SHEAR TEST		TRIAXIAL SHEAR TEST σ_3=2 & 2.5 Kg/cm^2 UNCONSOLIDATED UNDRAINED		CONSOLIDATED UNDRAINED		CONSOLIDATED DRAINED	
			C	ϕ	C	ϕ	C	ϕ	C	ϕ
AT	1.56	25	0.11	23	0.33	0	0.1	6	0.1	7
SECTION	1.68	21	0.00	33	0.10	3	0.0	10	-	-
1-1	1.68	20	0.15	29	0.50	0	0.1	5.5	0.1	9
AT	1.73	21	0.00	35.0	0.75	0	0.1	10	0.10	9.5
SECTION	1.65	26	0.10	19.0	0.40	1	0.1	5	0.15	6.0
2-2	1.57	27	0.10	26.5	0.45	0	0.1	6	0.10	7.0
AT	1.65	22	0.12	27	0.35	0	0.10	10	0.05	13
SECTION	1.60	24	0.08	29	0.30	0	0.15	7	0.1	8
3-3	1.50	26	0.80	20	0.25	3	0.10	5	0.1	8

C in kg/cm^2 and ϕ in deg

TABLE - 1.B

RESULTS OF SHEAR PARAMETERS C & ϕ

S.No.	Location of Sample	Density (gm/cc) Dry	Wet	Moisture Content (in %)	Direct Shear Test Cohesion (kg/cm^2)	ϕ deg	Triaxial Shear Test Cohesion (kg/cm^2)	ϕ deg
1	AT	1.49	1.84	23	0.09	29	0.018	10
2	SECTION	1.65	2.00	21	0.12	27	0.080	17
3	3-3	1.55	1.90	22	0.11	25	--	-
1	AT	1.49	1.88	26	0.10	23	0.110	11
2	SECTION	1.42	1.82	28	0.05	28	0.060	15
3	4-4	1.61	1.98	23	0.09	29	--	-

NOTE : a) Sample were tested after 3 days soacking in water.
b) ϕ is Angle of Internal Friction.

The variation in the results is very wide and inspite of getting the test done recently it's difficult to arrive at some reasonable values of shear parameters. The direct shear test being simple could be considered more reliable, give the indication that the ϕ of soil is about 25°. However, the triaxial tests indicated it to be not more than 15°. Generally the difference between the triaxial test and direct shear test value is only of the order of 1° to 2°. However, it can be inferred from both the test that cohesion in the soil is of very small magnitude. With these results, there seems to be no other choice with the designer except to go for back analysis to evaluate the soil shear parameters for design of safe slopes.

THE STUDY

Since the properties of soil varied untenably over a wide range and keeping in view the shear surfaces as observed at site the study consists of the following:

A. Analysis By Slip Circle Method:

1. Performing back analysis on slopes existing prior to earthquake under the following conditions on all four sections to determine the most likely soil properties of the site. (Chaturvedi and Lavania, 1970)

a) Horizontal seismic coefficient as 0.0, 0.05 and 0.1 at the instant of failure of slope,
b) Pore pressure as 0%, 10% and 20% of the weight of material,
c) Factor of safety has been assumed to be 1.0 under pseudo static condition.

B. Analysis Of Slopes Using Finite Element Method

It involves three steps:

i) Determination of major principal strains for sections 1-1 & 3-3.

ii) Location of potential failure surface using major principal strain contours, and

iii) Comparing factor of safety computed by finite element method with conventional slip circle method.

C. Design Of Safe Slope

i) Designing safe slope from the sets of soil parameters so evaluated.
ii) Recommendations for the safe slopes.

BACK ANALYSIS

Back analysis of the slope with the knowledge of failure shear surface, provides reasonably good basis for assessing the soil shear parameters in cases where a wide variation in test results is observed.

As these slides took place during earthquake there must be some contribution of seismic inertia force in the initiation of slide. Also, slides took place during rainy season so pore pressure may play a significant role. Data show that dry density of soil at site is 1.63 gm/cc but the moisture content of the soil was 23%. As the moisture content of 23% almost saturates the soil so saturated density of 2.0 gm/cc was taken for the analysis to get the results on conservative side.

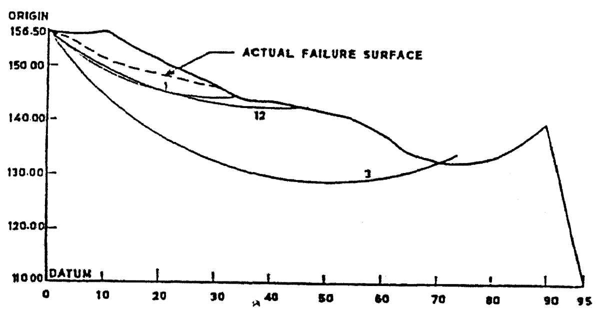

Fig 2. ORIGINAL PROFILE 1-1 SHOWING SHEAR SURFACES

For section 1-1, the actual failure surface was at a shallow depth (fig. 2). So only such combinations of soil shear parameters (ϕ & C) were selected which gave critical surfaces close to the actual failure surface. For that, combination of C-ϕ selected are such that these have smaller value of cohesion 'C', i.e., which correspond to failure surfaces 12 and 1 (fig. 2). So with the help of back analysis, range of ϕ selected is 20° to 24°. The possible combinations of C-ϕ are shown below in Table-2.

For section 2-2 and 4-4, the actual failure surface are shown in fig. 3 & 5 respectively. Thus from the various combinations of C and ϕ obtained through back analysis, only such combinations were selected which have cohesion 'C' equal to zero and 'ϕ' such that critical surface corresponds to actual failure surface. However, as the slides are surface slide so no pore water pressure will be in this case.

For section 2-2, range of ϕ selected is 22° to 28°. The possible combinations obtained are $\phi = 22^\circ$, C = 0.0 t/m^2 (for α_h = 0.0); $\phi = 25^\circ$, C = 0 t/m^2 (for α_h = 0.05); and $\phi = 28^\circ$, C = 0.0 t/m^2 for (α_h = 0.1) for no pore water pressure.

TABLE - 2

	U = 0.0		U = 0.1	
α_h	ϕ (deg)	C (t/m^2)	ϕ (deg)	C (t/m^2)
0.0	18	0.40	18	0.90
0.0	20	0.20	20	0.50
0.05	21	0.40	21	0.80
0.05	22	0.25	22	0.60
0.1	22	0.75	22	1.40
0.1	24	0.40	24	0.95

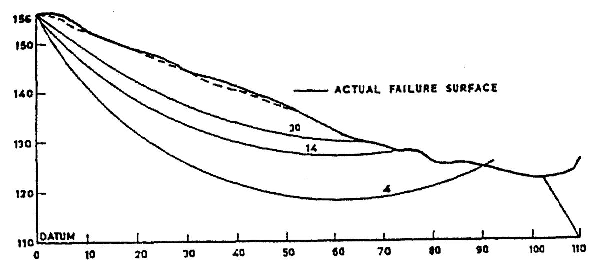

Fig 3. ORIGINAL PROFILE 2-2 SHOWING SHEAR SURFACES

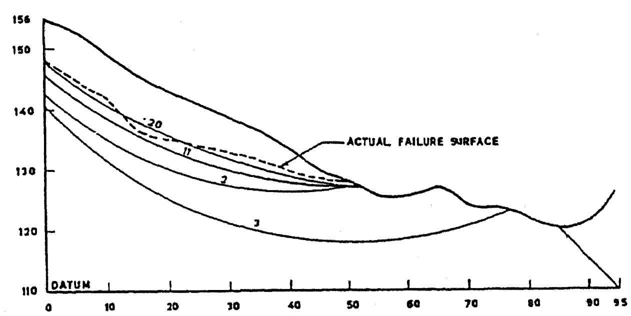

Fig 4. ORIGINAL PROFILE 3-3 SHOWING SHEAR SURFACES

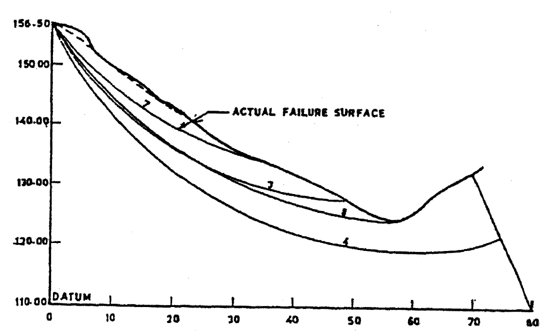

Fig 5. ORIGINAL PROFILE 4-4 SHOWING SHEAR SURFACES

Similarly, for section 4-4, range of ϕ selected is $32°$ to $38°$. The possible combinations obtained are $\phi = 32°$, $C = 0.0$ t/m^2 (for $\alpha_h = 0.0$); $\phi = 35°$, $C = 0.0$ t/m^2 (for $\alpha_h = 0.05$); & $\phi = 38°$, $C = 0.0$ t/m^2 (for $\alpha_h = 0.1$), for no pore water pressure condition.

For section 3-3, it may be seen that the actual failure surface was at a shallow depth (fig. 4). However, as the slides have occurred at certain depth, the soil is expected to have a little cohesion. Thus only such combinations of C-ϕ are selected which give critical surface close to the actual failure surface. For that, such combinations of C-ϕ were selected which correspond to critical surface 11 and 20 (shown in fig. 4). The possible combinations of C-ϕ are shown in Table - 3

TABLE - 3

α_h	U = 0.0		U = 0.1	
	ϕ (deg)	C (t/m^2)	ϕ (deg)	C (t/m^2)
0.00	22	0.80	22	1.50
0.00	23	0.60	23	1.25
0.05	23	1.35	23	2.10
0.05	25	0.90	25	1.50
0.1	27	1.15	27	1.90
0.1	28	0.90	28	1.60

ANALYSIS BY FINITE ELEMENT METHOD

A linear elastic FEM analysis considering the soil slope as plane strain problem, was carried out to determine the probable shear surface under normal (non-earthquake) condition for the purpose of comparing this with the actual one. Also the safety factors obtained by FEM analysis using the approach given by Rosendiz and Rome (1972), have been compared with that from conventional slip circle analysis. The sections 1-1 and 3-3 as existing prior to earthquake have been discretized using four noded quadrilateral elements. The base of slope as well as hill side of slope have been treated as fixed. To avoid the effect of stress concentration due to the assumption of fixed boundaries, sections were extended on both sides as well as in the direction of depth. Poisson's ratio is taken as 0.35 and shear wave velocity as $V_s = 100$ m/sec.

Potential failure surfaces defined by the contours of major principal strains along a continuous band of strain concentration are located for section 1-1 and section 3-3 as shown in fig. 6 and 7 respectively.

The factor of safety for the embankment slope was obtained by finite element model by determining the average value of $R=\sigma_{df}/\sigma_d$ along the critical surface defined by the line of maximum principal strain, where σ_d and σ_{df} represent the deviatoric stress at any point on the critical surface and the deviatoric stress at failure respectively. This ratio (R) also helps in assessing the accuracy of soil shear parameters obtained by back analysis using slip circle method. If ratio R is less than unity it indicates that value of C and ϕ considered are on conservative side Hence, only those combinations

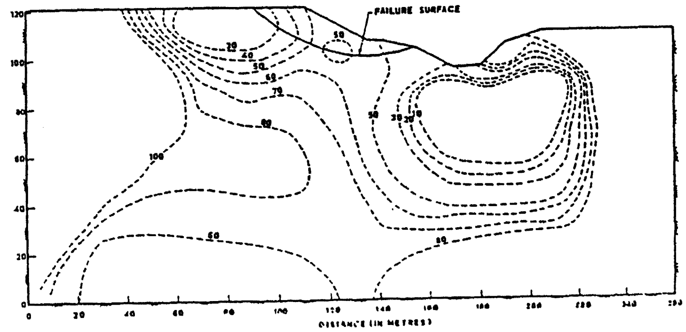

Fig 6. MAJOR PRINCIPAL STRAIN CONTOURS FOR SECTION 1-1

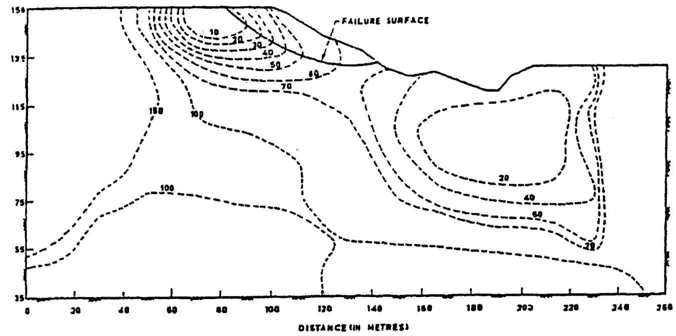

Fig 7. MAJOR PRINCIPAL STRAIN CONTOURS FOR SECTION 3-3

of C and ϕ are used in design of slope which give ratio R less than or equal to unity.

For section 1-1, combinations of soil shear parameters ϕ and C selected are $22°$, 0.25 t/m^2; $22°$, 0.6 t/m^2; and $24°$, 0.4 t/m^2. The ratios R obtained for these combinations are 0.896, 0.959 and 1.04 respectively as shown in Table -4. Thus it indicates that value of ϕ and C to be used in design of slope should be around $22°$, 0.25 t/m^2 or $22°$, 0.6 t/m^2. On the other hand for section 3-3, combinations of ϕ, C selected are $25°$, 0.9 t/m^2, $25°$, 1.5 t/m^2; and $28°$, 0.9 t/m^2. The ratio obtained for these combinations are 0.847, 0.922 and 1.01 respectively (Table - 4). Thus it indicates that value of ϕ, C to be used in design of slopes should be $25°$, 0.9 t/m^2 or $25°$, 1.5 t/m^2, which are also justified by slip circle method.

DESIGN OF SAFE SLOPES

A number of alternatives, e.g., provision of boulder facing, short concrete piles etc. were considered, but the most economical and safe was found to be the flattening of slopes by filling with locally available soil.For section 1-1, safe slopes have been designed for the combination of C-ϕ shown in Table- 1. The slopes are designed for $\alpha_h = 0.15$ and $\alpha_h = 0.2$ taking pore water pressure as 10% of weight of soil. Safe slopes for various combinations of shear parameters of 1-1,obtained for above mentioned conditions, are given in Table-5.

TABLE - 4

Section	$C(t.m^2)$	ϕ(deg)	V=(m/s)	$(\sigma_1-\sigma_3)_{FEM}$	$(\sigma_1-\sigma_3)_{df}$	$R = \frac{\sigma_{df}}{\sigma_{FEM}}$
	0.25	22	100.0	-16.319	-14.615	0.896
1-1	0.60	22	100.0	-16.319	-15.653	0.959
	0.40	24	100.0	-16.319	-17.109	1.040
	0.90	25	100.0	-25.483	-21.596	0.847
3-3	1.50	25	100.0	-25.483	-23.478	0.922
	0.90	28	100.0	-25.483	-25.688	1.010

TABLE - 5

S.No.	ϕ(deg)	C (t/m^2)	α_h	Safe Slope (H:V)
1	20	0.20	0.15	5.3 : 1
2	20	0.20	0.20	7.3 : 1
3	22	0.25	0.15	4.4 : 1
4	22	0.25	0.20	6.1 : 1
5	22	0.60	0.15	4.0 : 1
6	22	0.60	0.20	5.2 : 1
7	24	0.40	0.15	3.7 : 1
8	24	0.40	0.20	4.6 : 1
9	24	0.95	0.15	3.4 : 1
10	24	0.95	0.20	4.1 : 1

Thus, the safe slope, with $\phi = 22°$, $C = 0.25$ t/m^2 of the soil, corresponding to $\alpha_h = 0.15$ and $\alpha_h = 0.2$ would be 4.4 : 1 & 6.1 : 1. The safe slope with $\phi = 22°$, $C = 0.6$ t/m^2 of the soil corresponding to these horizontal seismic coefficients would be 4 : 1 & 5.2 : 1. This shows that slope value vary in range of 4 : 1 to 6.1 : 1 for the selected shear parameter of section 1-1. Taking an average of the slope i.e., 5. : 1 (H:V) slope would provide safety for $\alpha_h = 0.17$ for section 1-1. The resons for using designed horizontal seismic coefficient as 0.17 is that the Nagaland site is situated in seismic zone V and the basic horizontal seismic coefficient for zone V is 0.08 (IS:1893-1984). Taking importance factor as 2.0, the value of desig horizontal seismic coefficient becomes $\alpha_h = 0.08 * 2 = 0.16$.

As the slopes of sections 2-2, 3-3 and 4-4 are also not safe for future earthquake condition, considering α_h equal to 0.15 & 0.20, the slopes at these sections would require filling to a flatter slope. This fill material would be nearby existing soil which has soil parameters as evaluated for section 1-1. Thus it can be inferred that the filling of the soil, having the shear parameters evaluated for section 1-1 would require the same slope at all the four section locations. Thus the safe slope to be provided for section 2-2, 3-3 and 4-4 is governed by the properties of fill material. Hence the proposed safe slope for other sections also is 5 : 1 (H:V).

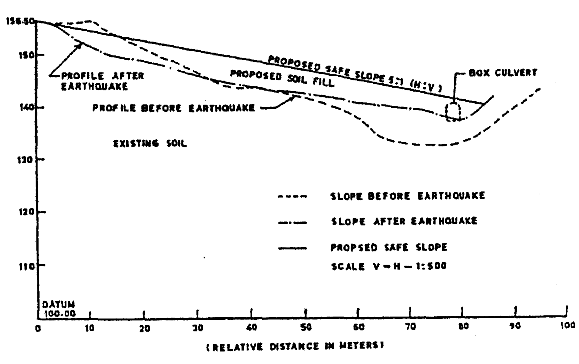

Fig 8. PROPOSED SAFE SLOPE FOR SECTION 1-1

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